5.4.2 Normal and Structural Low-Density Concrete

5.4.2.1 COMPRESSIVE STRENGTH

Add a new sentence to the end of paragraph three as follows:

The specified compressive strength for reinforced concrete shall not be less than 3.60 ksi (25Mpa).

C5.4.2

C5.4.2.1

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5.5.4 Strength Limit State
5.5.4.1 GENERAL
5.5.4.2 RESISTANCE FACTORS

5.5.4.2.1 Conventional Construction
Add a new bullet after the first bullet as follows:

- For tension-controlled cast-in-place prestressed concrete sections as defined in Article 5.7.2.1……………………..0.95

C5.5.4
C5.5.4.1
C5.5.4.2

C5.5.4.2.1
Delete Fig. C5.5.4.2.1-1 and replace with the following:

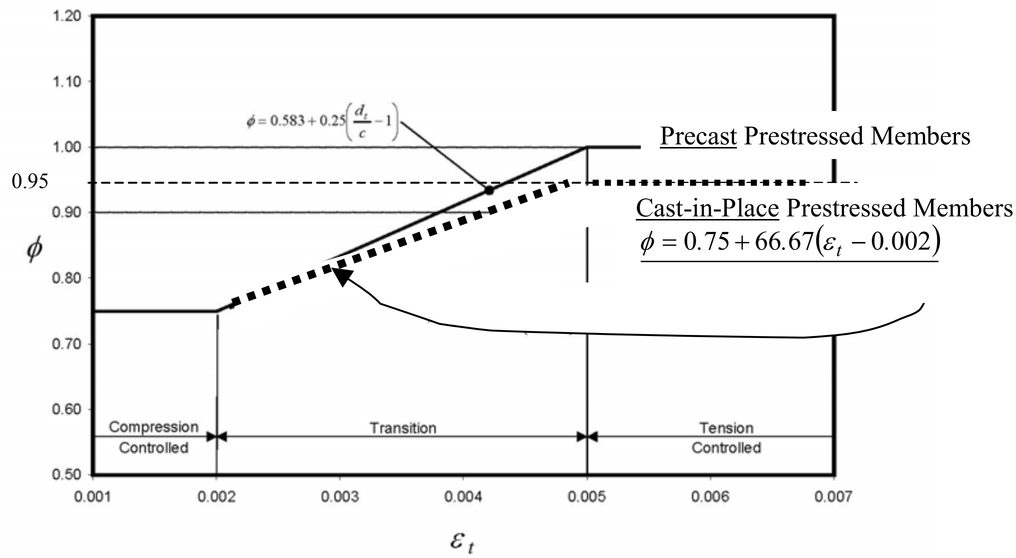

Figure C5.5.4.2.1-1 – Variation of ϕ with net tensile strain ε_t for ~~Grade 60 reinforcement and for~~ prestressed members.

Add Fig. C5.5.4.2.1-2 as follows:

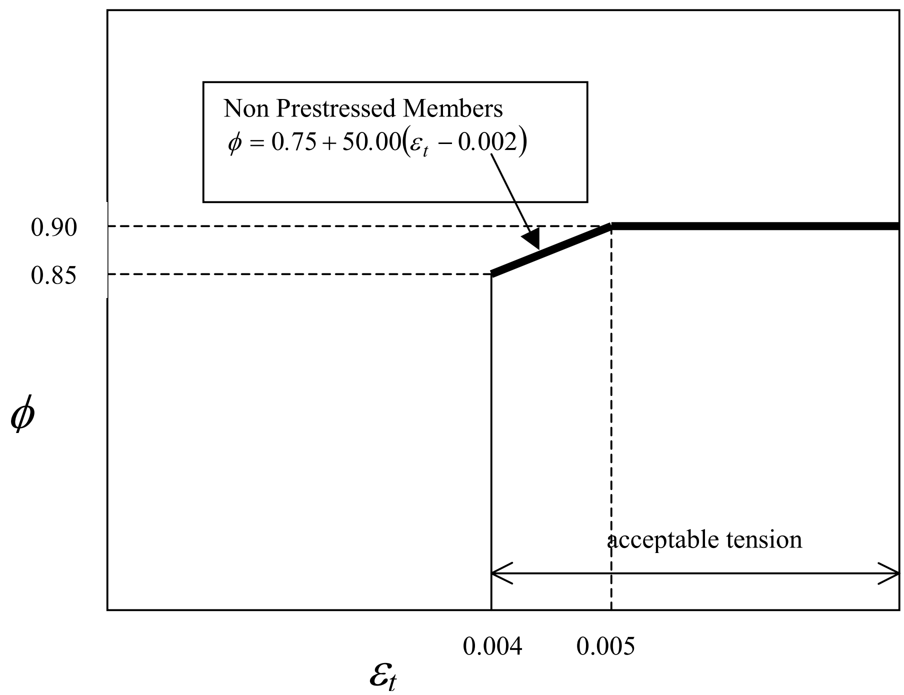

Figure C5.5.4.2.1-2 – Variation of ϕ with net tensile strain ε_t for Grade 60 reinforcement conventionally reinforced concrete members.

5.5.5. Extreme Event Limit State
Revise as follows:

The structure as a whole and its components shall be proportioned to resist collapse due to extreme events, specified in Table 3.4.1-1, as may be appropriate to its site and use. Resistance factors shall be 1.0.

C5.5.5

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5.7.2.1 GENERAL

Add a new bulleted item as follows:

- In the approximate flexural resistance equations of Articles 5.7.3.1 and 5.7.3.2, f_y and f'_y may replace f_s and f'_s, respectively, subject to the following conditions:
  - f_y may replace f_s when, using f_y in the calculation, the resulting ratio c/d_s does not exceed 0.6. If c/d_s exceeds 0.6, strain compatibility shall be used to determine the stress in the mild steel tension reinforcement.
  - f'_y may replace f'_s when, using f'_y in the calculation, $c \geq 3d'_s$. If $c < 3d'_s$, the compression reinforcement may be conservatively ignored ($A'_s = 0$), or strain compatibility shall be used to determine the stress in the mild steel compression reinforcement.

C5.7.2.1

Add new paragraphs as follows:

When using the approximate flexural resistance equations in Articles 5.7.3.1 and 5.7.3.2, it is important to assure that both the tension and compression mild steel reinforcement are yielding to obtain accurate results. In previous editions of AASHTO LRFD Bridge Design Specifications, the maximum reinforcement limit of $c/d_e \leq 0.42$ assured that the mild tension steel would yield at nominal flexural resistance, but this limit was eliminated in the 2006 Interim revisions. The current limit of $c/d_s \leq 0.6$ assures that the mild tension steel will be at or near yield, while $c \geq 3d'_s$ assures that the mild compression steel will yield. It is conservative to ignore the compression steel when calculating flexural resistance. In cases where either the tension or compression steel does not yield, it is more accurate to use a method based on the conditions of equilibrium and strain compatibility to determine the flexural resistance.

The mild steel tension reinforcement limitation does not apply to prestressing steel used as tension reinforcement. The equations used to determine the stress in the prestressing steel at nominal flexural resistance already consider the effect of the depth to the neutral axis.

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5.7.3.1.1 Components with Bonded Tendons

C5.7.3.1.1

Delete Eq. 5.7.3.1.1-3 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{\underline{f_s}} - A'_s \underline{\underline{f'_s}} - 0.85 f'_c \left(b - b_w\right) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Delete Eq. 5.7.3.1.1-4 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{\underline{f_s}} - A'_s \underline{\underline{f'_s}}}{0.85 f'_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Delete the definitions for f_y and f'_y and replace with the following:

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

Delete the last paragraph of Article 5.7.3.1.1:

~~The stress level in the compressive reinforcement shall be investigated, and if the compressive reinforcement has not yielded, the actual stress shall be used in Eq. 3 instead of f'_y.~~

5.7.3.1.2 Components with Unbonded Tendons

C5.7.3.1.2

Delete Eq. 5.7.3.1.2-3 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{\underline{f_s}} - A'_s \underline{\underline{f'_s}} - 0.85 f'_c \left(b - b_w\right) h_f}{0.85 f'_c \beta_1 b_w}$$

Delete Eq. 5.7.3.1.2-4 and replace with the following:

$$c = \frac{A_{ps} f_{pu} + A_s \underline{\underline{f_s}} - A'_s \underline{\underline{f'_s}}}{0.85 f'_c \beta_1 b}$$

Delete the last paragraph:

~~The stress level in the compressive reinforcement shall be investigated, and if the compressive reinforcement has not yielded, the actual stress shall be used in Eq. 3 instead of f'_y.~~

5.7.3.2.2 Flanged Sections

Delete Eq. 5.7.3.2.2-1 and replace as follows:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s \underline{\underline{f_s}}\left(d_s - \frac{a}{2}\right) - A_s{}' \underline{\underline{f'_s}}\left(d_s{}' - \frac{a}{2}\right) + 0.85 f'_c \left(b - b_w\right) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Delete the definitions for f_y and f'_y and replace with following:

<u>f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1</u>

<u>f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1</u>

C5.7.3.2.2

5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

Delete Eq. 5.7.3.4-1 and replace as follows:

$$s \leq \frac{700\gamma_c}{\beta_s \underline{\underline{f_{ss}}}} - 2d_c$$

Revise the 3rd paragraph as follows:

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. Class 2 exposure condition applies to <u>all bridge decks</u>, ~~transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength~~ and when there is increased concern of appearance and/or corrosion. <u>The concrete cover d_c shall be taken as 2-1/2-in.</u>

C5.7.3.4

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5.7.3.6.2 Deflection and Camber

Revise the first paragraph and add a second paragraph as follows:

Instantaneous d~~D~~eflection ~~and camber~~ calculations shall consider appropriate combinations of dead load, live load, prestressing forces, erection loads, concrete creep and shrinkage, and steel relaxation.

Long-term deflection calculations to estimate camber shall consider deflections due to appropriate combinations of all the above mentioned load effects except for those due to live load.

C5.7.3.6.2

Add new text before paragraph one as follows:

"Camber" is the deflection built into a member, other than by prestressing, in order to achieve a desired grade.

Delete the current fifth paragraph and replace with the following:

~~Unless a more exact determination is made, the long-time deflection may be taken as the instantaneous deflection multiplied by the following factor:~~

- ~~If the instantaneous deflection is based on I_g: 4.0~~
- ~~If the instantaneous deflection is based on I_e: 3.0-1.2(A'_s/A_s) >=1.6~~

Long-term deflection of cast-in-place structures may be calculated by multiplying the instantaneous deflection values based on I_g with the following factors:

- For nonprestressed concrete structures: 4.0
- For prestressed concrete structures: 3.0

Alternatively, long-term deflection of cast-in-place non-prestressed concrete structures may be calculated by multiplying the instantaneous deflection values based on I_e with the following factor:

$$3.0 - 1.2(A'_g/A_s) \geq 1.6 \quad (5.7.3.6.2\text{-}3)$$

where:

A'_s = area of compression reinforcement (in^2)

A_s = area of nonprestressed tension reinforcement (in^2)

Add Commentary to Equation 1 as follows:

Past experiences with cast-in-place box girder bridges show that the design predictions of camber based on I_g are generally in conformance with field measured values.

Revise paragraph one as follows:

In prestressed concrete, the long-term deflection ~~is usually~~ may be based on mix-specific data where available, possibly in combination with the calculation procedures in Article 5.4.2.3. Other methods of calculating deflections which consider the different types of loads and the sections to which they are applied, such as that found in (*PCI 1992*), may also be used.

$$3.0 - 1.2(A'_g/A_s) \geq 1.6 \qquad (5.7.3.6.2\text{-}3)$$

5.8.2.9 SHEAR STRESS ON CONCRETE

Revise paragraph two as follows:

In determining the web width at a particular level, one-half the diameters of ungrouted ducts or one-quarter the diameter of grouted ducts at that level shall be subtracted from the web width. It is not necessary to reduce b_v for the presence of ducts in fully grouted cast-in-place box girder frames.

C5.8.2.9

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5.8.3.4.2 General Procedure

C5.8.3.4.2

Add text as follows:

Conservatively, non-concurrent values for M_u and V_u may be used to evaluate ε_t. When coincident values are used, both maximum M_u with coincident V_u, and maximum V_u with coincident M_u, should be checked. If approximate methods are used for distribution of live loads, the girder distribution factor for bending should be used for both maximum M_{LL} and coincident V_{LL} and vice versa. For Strength I, force effects due to both the typical and contraflexure truck configurations should be evaluated.

Add a new Article as follows:

5.8.3.4.3 Simplified Procedure

In lieu of Table 5.8.3.4.2-1, β and θ may be evaluated as follows for vertical stirrups:

$$\beta = \frac{4.8}{1 + 1500\varepsilon_x} \quad (5.8.3.4.3\text{-}1)$$

$$\theta = 29 + 7000\varepsilon_x \quad (5.8.3.4.3\text{-}2)$$

C5.8.3.4.3

Equations 1 and 2 were developed by Micheal Collins and adopted into the Canadian Specs (2004). The effect on resulting values for concrete shear resistance was found to be somewhat greater than the sectional method, but less than those per the 2002 *AASHTO Standard Specifications* on which *Caltrans BDS* is based.

5.8.3.5 LONGITUDINAL REINFORCEMENT

C5.8.3.5

Add text as follows:

Conservatively, non-concurrent values for M_u and V_u may be used to evaluate longitudinal reinforcement. When coincident values are used, both maximum M_u with coincident V_u, and maximum V_u with coincident M_u, should be checked. If approximate methods are used for distribution of live loads, the girder distribution factor for bending should be used for both maximum M_{LL} and coincident V_{LL} and vice versa. For Strength I, force effects due to both the typical and contraflexure truck configurations should be evaluated.

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5.9.3 Stress Limitations for Prestressing Tendons

C5.9.3

Revise Table 5.9.3-1 as follows:

~~Prior to Seating~~	$0.90f_{py}$	~~0.90fpy~~	$0.90f_{py}$
Maximum Jacking Stress	$0.90f_{py}$	$0.75f_{pu}$ (see note)	$0.90f_{py}$

Add a note below Table 5.9.3-1 as follows:

Note: For longer frame structures, tensioning to $0.90f_{py}$ for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value (low relaxation strand, only).

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5.9.4.2.2 Tension Stresses

Revise Table 5.9.4.2.2-1 for "other than segmentally constructed bridges" as follows.

C5.9.4.2.2

Table 5.9.4.2.2-1 Tensile Stress Limits in Prestressed concrete at Service Limit State After Losses, Fully Prestressed Components

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections • For components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only. • For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions, and are located in Caltrans Environmental Areas I or II. • For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, and are located in Caltrans Environmental Area III. • For components with unbonded prestressing tendons.	No tension $0.19\sqrt{f'c}$ (ksi) $0.0948\sqrt{f'c}$ (ksi) No tension

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5.9.5.2.3b Posttensioned Members

Revise Table 5.9.5.2.3b as follows:

Type of Steel	Type of Duct	K (1/ft)	μ
Wire or strand	Rigid and semi-rigid galvanized metal sheathing Tendon Length:		~~0.15-0.25~~
	≤ 600 ft	0.0002	0.15
	600 ft ≤ 900 ft	0.0002	0.20
	900 ft ≤ 1200 ft	0.0002	0.25
	> 1200 ft	0.0002	>0.25
	Polyethylene	0.0002	0.23
	Rigid steel pipe deviators for external tendons	0.0002	0.25
HS bars	Galvanized metal sheathing	0.0002	0.30

C5.9.5.2.3

Add new last paragraph as follows:

For tendon lengths greater than 1200 feet, investigation is warranted on current field data of similar length frame bridges for appropriate values of μ.

5.9.5.2.3b

Delete Equation 5.9.5.2.3b-1 and replace with the following:

$$\Delta f_{pES} = \underline{\underline{0.50}} * \frac{E_p}{E_{ci}} f_{cgp}$$

C5.9.5.2.3b

Delete Equation C5.9.5.2.3b-1 and replace with the following:

$$\Delta f_{pES} = \underline{\underline{0.50}} * \frac{A_{ps} f_{pbt} \left(I_g + e_m^2 A_g\right) - e_m M_g A_g}{A_{ps}\left(I_g + e_m^2 A_g\right) + \frac{A_g I_g E_{ci}}{E_p}}$$

.

5.9.5.3 APPROXIMATE ESTIMATE OF TIME-DEPENDENT LOSSES

Add text to the end of Article 5.9.5.3 as follows:

For ~~cast-in-place~~ post-tensioned members, the approximate estimate of time-dependent losses may be taken as the lump sum value of 25 ksi.

C5.9.5.3

Add text to the end of Article C5.9.5.3 as follows:

The expressions for estimating time-dependent losses in Table 5.9.5.3-1 were developed for pretentioned ~~precast~~ members and should not be used for ~~cast-in-place~~ post-tensioned structures. Preliminary research at UCSD indicates that the time-dependent losses for cast-in-place post-tensioned structures are between 25 ksi and 30 ksi. Until the research is completed, and, in lieu of a more detailed analysis, a lump sum value for losses in post-tensioned members is provided.

Table 5.9.5.3-1 Time-Dependent Losses in ksi.

Type of Beam Section	Level	For Wires and Strands with f_{pu} = 235, 250 or 270 ksi	For Bars with f_{pu} = 145 or 160 ksi
Rectangular Beams, Solid Slab	Upper Bound Average	29.0 + 4.0 *PPR* 26.0 + 4.0 *PPR*	19.0 + 6.0 *PPR*
Pretensioned Box Girder	Upper Bound Average	21.0 + 4.0 *PPR* 19.0 + 4.0 *PPR*	15.0
Single T, Double T, Hollow Core and Voided Slab	Upper Bound Average	$39.0\left[1.0 - 0.15\frac{f'_c - 6.0}{6.0}\right] + 6.0\,PPR$ $33.0\left[1.0 - 0.15\frac{f'_c - 6.0}{6.0}\right] + 6.0\,PPR$	$31.0\left[1.0 - 0.15\frac{f'_c - 6.0}{6.0}\right] + 6.0\,PPR$

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5.11.4.3 PARTIALLY DEBONDED STRANDS
Revise paragraphs two and three as follows:

The number of partially debonded strands should not exceed ~~25~~ 33 percent of the total number of strands.

The number of debonded strands in any horizontal row shall not exceed ~~40~~ 50 percent of the strands in that row.

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